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Foundation design study,  
Pier 169, Proposed Astoria

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FOUNDATION DESIGN STUDY

PIER 169  
PROPOSED ASTORIA BRIDGE  
ASTORIA, OREGON

FOR THE

OREGON STATE HIGHWAY DEPARTMENT

# DAMES & MOORE

CONSULTANTS IN APPLIED EARTH SCIENCES  
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PARTNER: IRVING E. OLSEN

January 26, 1962

State of Oregon  
State Highway Department  
Salem 10, Oregon

Attention: Mr. Forrest Cooper, State Highway Engineer

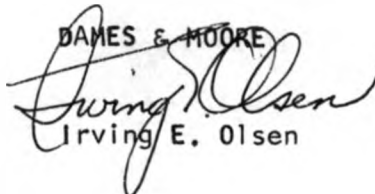
Gentlemen:

We enclose herewith six copies of our report "Foundation Design Study, Pier 169, Proposed Astoria Bridge, Astoria, Oregon for the Oregon State Highway Department".

The scope of the investigation was defined in our letter of proposal dated January 8, 1962. The design study was authorized by your contract dated January 12, 1962.

The preliminary results of our studies and our preliminary recommendations were discussed with various representatives of your office in a meeting in Salem on January 18. Our studies indicate that support of the proposed Pier 169 on relatively short piling according to your preliminary design is feasible only if moderate settlements of the pier can be tolerated. Support of the pier on longer piling would reduce settlements.

Yours very truly,

DAMES & MOORE  
  
Irving E. Olsen

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FOUNDATION DESIGN STUDY  
PIER 169  
PROPOSED ASTORIA BRIDGE  
ASTORIA, OREGON  
FOR THE  
OREGON STATE HIGHWAY DEPARTMENT

INTRODUCTION

400-1114  
The proposed Astoria bridge will span the Columbia River at Astoria, Oregon and will connect Astoria with Megler, Washington. The proposed bridge will be over four miles in length. The main span of the bridge will consist of a pair of parallel rigid steel trusses supported on two main concrete piers. The distance between the centers of the piers will be 1232 feet. The northernmost of these two piers, Pier 169, is the subject of this report.

The Oregon State Highway Department has conducted extensive field explorations to determine the soil conditions at the locations of the two main piers and at many other locations along the bridge alignment. These explorations reveal that bed rock exists at an economical depth beneath the location of Pier 170, the southernmost main pier. Soft compressible strata were encountered to considerable depths beneath the soil surface at the location of Pier 169. Differential settlements between the two piers are to be held to a minimum. The purpose of our study of Pier 169 is to analyze the design of the pier utilizing field and laboratory findings obtained by the State and to present recommendations for alternate methods of foundation support if the anticipated settlements are found to be excessive.

### PIER DESIGN CRITERIA

Pier 169 will consist of two identical concrete pedestals which will be designed to carry equal loads. At their bases, the pedestals will be approximately 57 feet by 106 feet in plan dimensions. The bases of the two pedestals will form a seal for the bottom of the cofferdam in which the main portions of the pedestals will be constructed. The seals will be some 16 feet in thickness with a base elevation of -65. Elevations used throughout this report refer to mean sea level.

4117-004  
Each half of the pier will weigh some 18,000 kips. Our computations indicate that this includes the effect of bouyancy on the submerged portion of the pier. Loads transferred to each half of the pier from the legs of the tower supporting the bridge will be 6,280 kips. Very nearly 100 percent of this load will be dead load. Our computations indicate that each half of the pier will impose an effective weight of approximately 7000 kips on its foundation during the period when the cofferdam is dry. This figure includes the effect of hydrostatic uplift on the base of the pier.

It is our understanding that it is presently proposed to support each half of the pier on 436 wood piles driven to a tip elevation of -115. The tops of the piles would extend far enough into the seal to insure complete bond for uplift. The pile spacing would vary and would be approximately three feet on centers along the outer edges of the pier and approximately six feet on centers in the central portion of the pier. The average effective load carried by each pile would be on the order of 56 kips.

We understand that the bridge will be a total of two lanes in width. The resulting narrow structure has required that the trusses forming the main span be rigid and continuous for lateral stability. Because of the structural continuity between the two main piers, differential deflections developed between the two piers will result in internal stresses within the truss members.

We have no information as to the amount of differential movement which can be tolerated. In our studies we have assumed that the critical settlement is that which would occur following completion of the span and that somewhat greater total settlement of the pier might be tolerated if this settlement is not transmitted to the superstructure.

#### SUBSURFACE CONDITIONS

400-6114  
Our description of subsurface conditions which follows is based on our interpretations of the data provided to us by the State of Oregon Highway Department. Since exploration, sampling and testing methods used at the bridge site differ somewhat from our own, we have attempted to convert the data into a more familiar form to facilitate our review and computations. The classification system to which our descriptions refer is presented on Plate A-1. The descriptions are highly generalized and are based mainly on our review of the State boring logs as modified by our observations of a limited number of disturbed soil samples. An idealized representation of the boring log data is presented on Plate A-2.

Three exploratory borings were drilled by the highway department at the location of Pier 169. The river bottom at the pier location is indicated to be at about elevation -48. Approximately the upper forty feet of material encountered in the borings consists of a fine to medium sand containing variable amounts of finer grained material. The sand is a river deposit and is evidently a layered system. The finer grained materials, consisting largely of silt, are also in layers. The sand stratum appears to be continuous to an elevation of approximately -90. There is little field evidence to indicate the characteristics of this material; undisturbed samples were not obtained. Blow counts on a sample of the sand obtained from an adjacent boring indicate that the material has very low resistance to penetration of the sampler. In addition, our experience would indicate that this sand is very probably quite loose in place, and

that the material would be generally of low strength with a low relative density.

Underlying the surface sand stratum is a continuous stratum of mixed silty soils extending at least to elevation -215. This stratum is apparently a highly complex layered system of silty loam, sandy loam and sand. Observations in the State laboratory indicate that the sand layers are extremely thin, although examination of the test data would indicate that some of the tests were performed on soils which were essentially all sand. It would therefore seem likely that there are at least occasional thicker layers of sand which may be horizontally continuous, and which might possibly serve as drainage paths for water squeezing out of finer grained soils.

Undisturbed samples were obtained from one of the borings throughout the thickness of the silty stratum. An extensive series of tests were performed on these soils. In general, these tests indicate that the soil is of moderately low strength and moderately high compressibility. The material possesses a fair degree of permeability and would consolidate rapidly under applied loads.

A 15-foot thick stratum of a material described on the State boring logs as a "clay loam" was encountered at elevation -215. Only a limited number of tests were performed on this material and the characteristics of the clayey soil have not been adequately determined. Field logs indicate the material to have a high rock content as well as organic matter, sand and silt.

At elevation -230 a compact stratum of rock fragments in a clayey matrix was encountered. The rock content of this material was observed to vary considerably. Field evidence indicates that this material is probably of high strength and density and is probably relatively incompressible. Laboratory data for the only undisturbed sample obtained of this is not conclusive and the sample is obviously not representative. Boring 13 was terminated in this stratum at elevation -272.

### INTERPRETATION OF TEST DATA

4117-004

The laboratory test program was apparently directed mainly toward determining the characteristics of the materials represented in the thick silt stratum. Although the samples probably represent a fairly wide range of soil types, the large number of each of the various types of tests performed apparently give a very good overall indication of the average characteristics of the stratum. Moisture and density determinations indicate that the material is of low density and correspondingly high moisture content. The densities are generally consistent with densities of similar soils encountered in the Columbia River bed at locations upstream. A graphical summary of most of the test data accumulated for the silty soils is presented on Plate A-2.

Strength test data were obtained by performing direct shear and unconfined and triaxial compression tests. The direct shear test data are somewhat erratic and tend to be distinctly inconsistent with the density data. The strengths obtained from the direct shear tests are uniformly much higher than those which would be expected from a silty soil of the indicated density. The triaxial and unconfined compression tests appear to be slightly more consistent with some exceptions and reveal, in general, lower strengths. In evaluating the supporting capacity of these soils we have assumed that the compression tests were all performed at the same lateral pressure, 40 pounds per square inch. A better evaluation of strengths would have resulted from a distribution of lateral pressures. The best average summary line of strengths which we were able to obtain by a Mohr-circle plot of the compression test data is a friction angle of 11 degrees and a cohesion of 300 pounds per square foot. These strength values are considerably lower than would be obtained by an overall averaging of the results of the compression tests and direct shear tests, but we believe that these values are probably far more indicative of the true strength of the soil. The strengths are peak values and are not related to either the yield or ultimate strengths of the material.

4117-004

The results of the consolidation tests indicate that the silty soils are moderately compressible and that the actual compression indices vary widely. These materials are relatively fast draining and a large percentage of the consolidation for each loading increment during the tests occurred prior to the first time reading. Apparently, compression of the test samples continued to occur over a fairly long period of time following the initial surge of compression. In general, the consolidation characteristics of these types of materials do not lend themselves to interpretation by conventional methods of analyses. The conventional theory is most readily applicable to slower draining clayey soils.

The bulk of the test data obtained for the "clay loam" between elevation -215 and -230 is obviously for a sandy soil. Apparently the material is highly variable and its behavior cannot be predicted with any assurance. We have assumed this material to have compressibility characteristics similar to the silty soils which overlie it. This assumption may introduce a substantial error in settlement predictions. However, the procedure is probably preferable to use of the data obtained for one sample of the material which is obviously not representative. This assumption has probably resulted in a small degree of conservatism with respect to prediction of ultimate settlement because of the indicated firmness of the material.

The characteristics of the lower compact material containing rock fragments were measured by a series of tests on one sample. We believe, in general, that this sample was not representative and that the tests are therefore of limited usefulness. The field data appear to indicate that the material is of reliably good strength and high density. Assuming the field data to be accurate, we would estimate that these materials are essentially incompressible and that they would offer excellent support to piling if piling were driven to that depth.



## DISCUSSION AND CONCLUSIONS

### General

400-6114

The results of our studies of the State boring logs and laboratory tests and our subsequent computations indicate that the use of relatively short friction piles will result in substantial settlements under the applied foundation loads. Because of certain inconsistencies in the laboratory test data and the lack of data for some of the soils, we believe that a moderate degree of conservatism is warranted in the foundation design. The figures which we present and discuss in the following paragraphs represent our best estimate of the probable conditions but are on the conservative side because of our general lack of knowledge of the true foundation conditions. The numerical results are not intended to be highly precise and should not be interpreted as such.

### Pile Support

The preliminary highway department design is for wood piles driven to a tip elevation of -115. We believe that piles of this length will be barely adequate to sustain the applied loads due to the action of the group in peripheral shear. According to our computations, a factor of safety of approximately 1.7 would exist for the piles against the peak supporting capacity of the soils. This value includes both the peripheral shear effects and the end bearing capacity of the group. Since a substantial portion of this capacity is obtained is obtained from the upper sand stratum, the characteristics of which are essentially unknown, we believe that a higher factor of safety against peak strengths is warranted. For this reason, we conclude that the piles should be driven deeper to safely sustain the applied loads. The use of higher capacity piling or a greater number of wood piles driven to this tip elevation would not increase the supporting capacity of the group.

We believe that a factor of safety of at least 2.0 should be used for the pile foundation design. On this basis, piles would have to extend to at least elevation -130. Assuming a pile cut-off at elevation -55, pile lengths would be on the order of 75 feet. Support at this level could be satisfactorily accomplished through the use of wood piles. Higher capacity piling would not substantially improve the supporting capacity of the group. Although piles driven to this depth would be satisfactory from a load carrying capacity standpoint, the resulting foundation may not be acceptable because of excessive settlement.

#### Settlements

In our computations we have assumed that only the settlements of the pier subsequent to the completion of the superstructure will be critical. Implicit in this assumption therefore is the fact that the pier may settle under its own weight by any reasonable amount without causing structural distress. By allowing the pier to remain in place for a substantial period of time before the superstructure is added, a large portion of the settlement of the pier under its own weight will have occurred prior to the time when settlements would be critical. We have studied the settlement problem as it relates to this important time consideration. Unfortunately, the rates at which settlements may be expected to occur for these soils is a highly indeterminate problem requiring a large degree of assumption. We believe that our time computations tend to be on the conservative side so that settlements measured in the field could very likely be more rapid than those which we have predicted from the results of the laboratory tests.

On Plate 1, we present a chart showing the relationship between various factors related to the settlement of the superstructure at the pier location. The factors considered in the chart are (1) time in months which the pier remains in place before the superstructure is added, (2) ultimate settlement of

the superstructure which is equal to settlement of the pier subsequent to the time the superstructure is added, (3) depth of piling and (4) factor of safety for the piling against ultimate load carrying capacity. Referring to the chart, a pile foundation with tips established at elevation -120 would undergo a settlement of approximately 14 inches if the pier and superstructure were constructed simultaneously. Recognizing that this is not a realistic consideration, the use of one of the upper curves would better illustrate field conditions. For instance, for pile tips established at elevation -120, eight inches of settlement would occur if the pier were allowed to remain in place for 12 months or one year prior to adding the superstructure.

One way to restrict settlements to slightly smaller magnitudes while using shorter piling might be to considerably widen the foundation, resulting in a generally modified foundation design, greater number of piling and associated costs. To widen the foundation, however, would be also to increase its weight. Since the foundation weight comprises a large percentage of the total weight of the structure, pressures on the compressible soils will not be substantially reduced by increasing the foundation size.

#### RECOMMENDATIONS

The chart on Plate 1 has been presented primarily for the purpose of showing the relationship between computed settlement and pile tip elevation. Other than reducing settlement, little advantage would result from deepening the piles below the depth required to support the piles in group shear. Assuming settlements of the general order of magnitude indicated on Plate 1 to be acceptable structurally, we recommend that the pier be supported on wood piles driven to elevation -130. These piles would be approximately 75 feet in length. We recommend that pile tip diameters be a minimum of seven inches.

We have studied the time-settlement characteristics of the pile group with pile tips established at -130. The results of these studies are presented

on Plate 2. The settlement at any time is shown with respect to the foundation load at that point of time. The loading diagram shown is for half-pier loads. The settlement remaining at any time may be found by subtracting the settlement anticipated up to that time from the estimated ultimate settlement of 12.4 inches. Assuming the major span to be completed, tying the two main piers together, at 20 months from time zero, the remaining settlement would be 12.4 minus 6.7, or 5.7 inches. This is the settlement which would induce stresses in the members of the main span truss. The remaining settlement of the pier may be computed in a similar manner for any other point in time.

It should be pointed out that the settlement figures presented in the foregoing paragraph and on Plates 1 and 2 do not represent a high order of precision. Settlements to the nearest tenth of an inch are not realistic and are shown as such merely for comparison purposes. Our settlement estimates are limited in accuracy by the general lack of certain soils data. This is particularly significant for the time rates where it is not known how much of an effect drainage through the minor sand layers will have on increasing the compression rate of the silty soils. It is believed, however, that the settlement estimates presented are as reasonable as might normally be expected for soils of the types considered.

In order to structurally compensate for settlement, we suggest that the pier be superelevated by approximately half the amount of estimated remaining settlement. For instance, at the time the pier is completed eight inches of additional settlement might be expected. Superelevating the pier by four inches would result in an eventual effective settlement from the design elevation of four inches. If the settlements proceed more rapidly than estimated, or if they are actually of somewhat lower magnitude than estimated, then the ultimate effective settlement of the pier would be less than the four inches. If significant deviations from our predictions do occur, we would expect that

they would result in settlements much more rapid than predicted. This effect would lead to less remaining settlement at any point in time than indicated on Plate 2.

We understand that alternate methods of pier support have not been entirely eliminated from consideration. If an alternate design to restrict settlement to a minimum is desired, we offer the following as a suggestion. To reduce settlement to very low magnitudes, we would recommend that steel H-piles be driven into the underlying dense strata below elevation -230.

Because of the greater capacity of the H-piles, the number of piles needed might be reduced by nearly two-thirds. The foundation size and weight would also be decreased, therefore further reducing the necessary number of H-piles. To widen the zone of pile influence, the outer two rows of H-piles might be driven on a slight batter.

The resulting foundation size might be of small enough proportions to permit the use of precast caisson sections which could be placed without the necessity of a cofferdam installation. A series of bell shaped caissons supported on a common base might be effectively used for support. Tremie concrete would be used to fill the precast forms.

It is believed that the precast pier installation together with H-piles might prove to be a highly satisfactory method of support, economically and with respect to bridge performance. It is suggested that this method be studied further if there is any possibility of redesign of the pier. Settlement of the H-pile group cannot be predicted with accuracy at this time, but it is certain that settlements would be substantially less than for a wood pile foundation established in the silt.

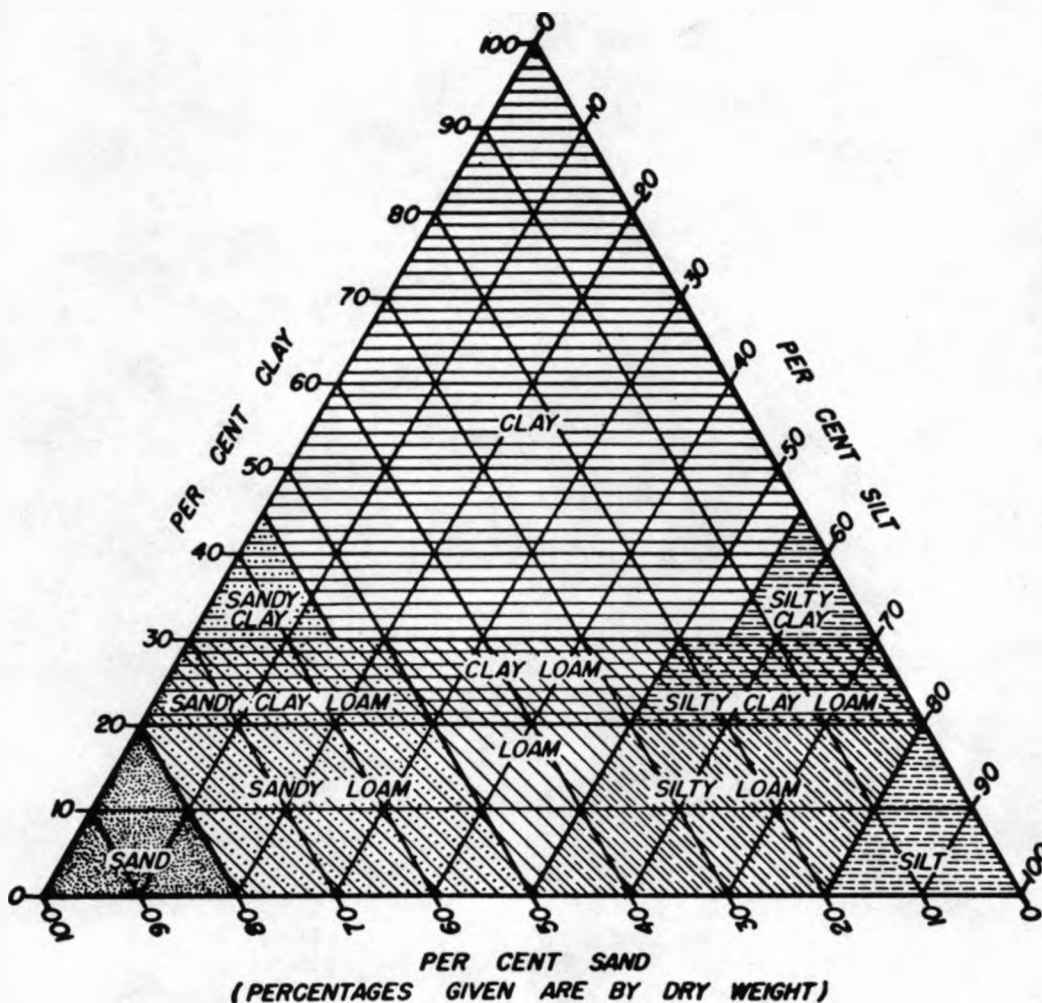
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*Irving S. Olsen*  
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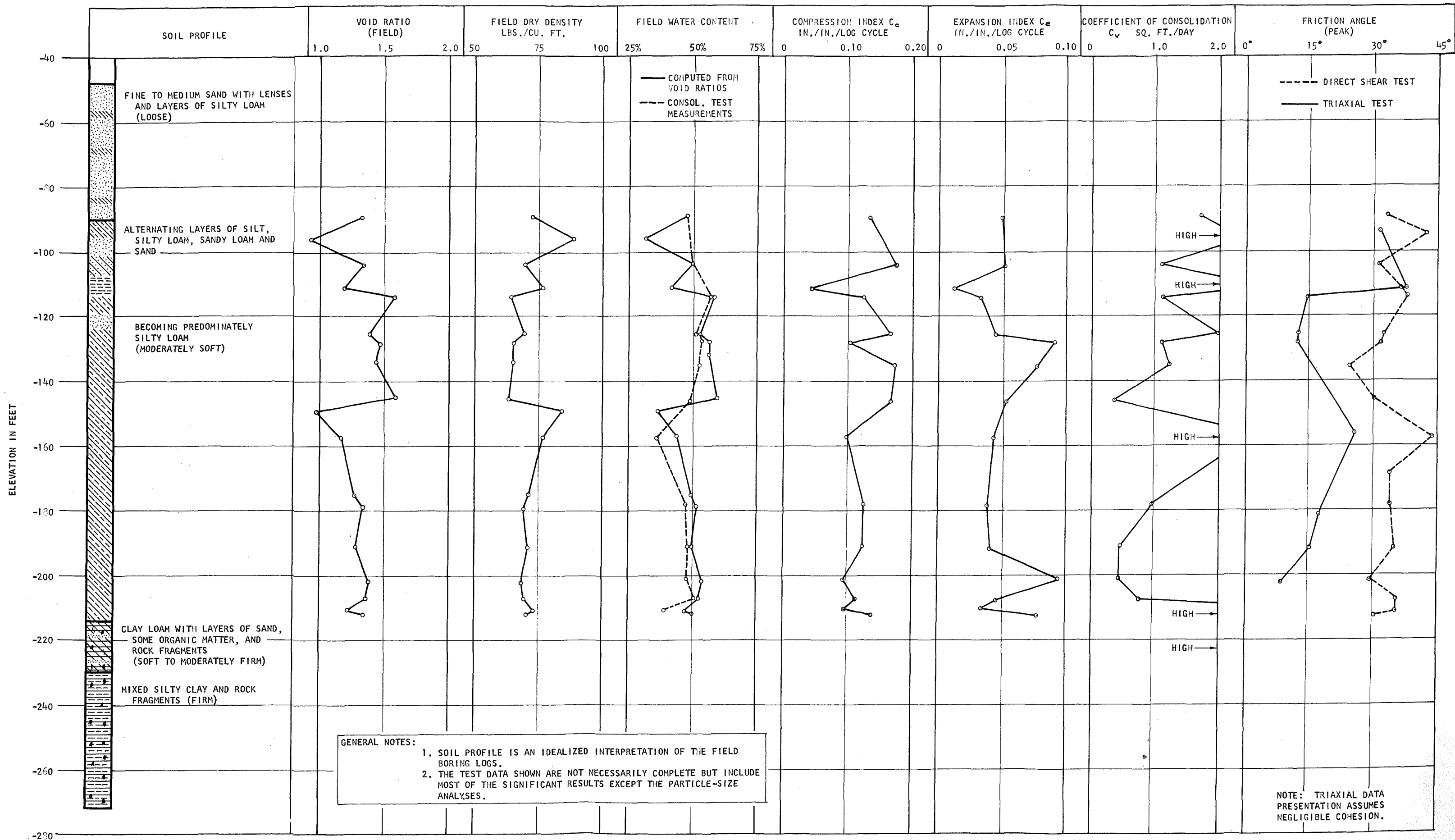
REVISIONS

BY \_\_\_\_\_ DATE \_\_\_\_\_  
 BY \_\_\_\_\_ DATE \_\_\_\_\_

| SOIL FRACTION | PARTICLE SIZE                                                          |        |             |        |
|---------------|------------------------------------------------------------------------|--------|-------------|--------|
|               | LOWER LIMIT                                                            |        | UPPER LIMIT |        |
|               | MM.                                                                    | INCHES | MM.         | INCHES |
| CLAY          |                                                                        |        | .005        | .0002  |
| SILT          | .005                                                                   | .0002  | .05         | .002   |
| SAND          |                                                                        |        |             |        |
| FINE          | .05                                                                    | .002   | .25         | .01    |
| MEDIUM        | .25                                                                    | .01    | .5          | .02    |
| COARSE        | .5                                                                     | .02    | 2.0         | .08    |
| ROCK FRACTION | THESE FRACTIONS ARE NOT CONSIDERED IN DETERMINING SOIL CLASSIFICATION. |        |             |        |
| GRAVEL        | 2.0                                                                    | .08    | 64.         | 2.5    |
| COBBLES       | 64.                                                                    | 2.5    | 256.        | 10.    |
| BOULDERS      | 256.                                                                   | 10.    |             |        |



SOIL CLASSIFICATION CHART



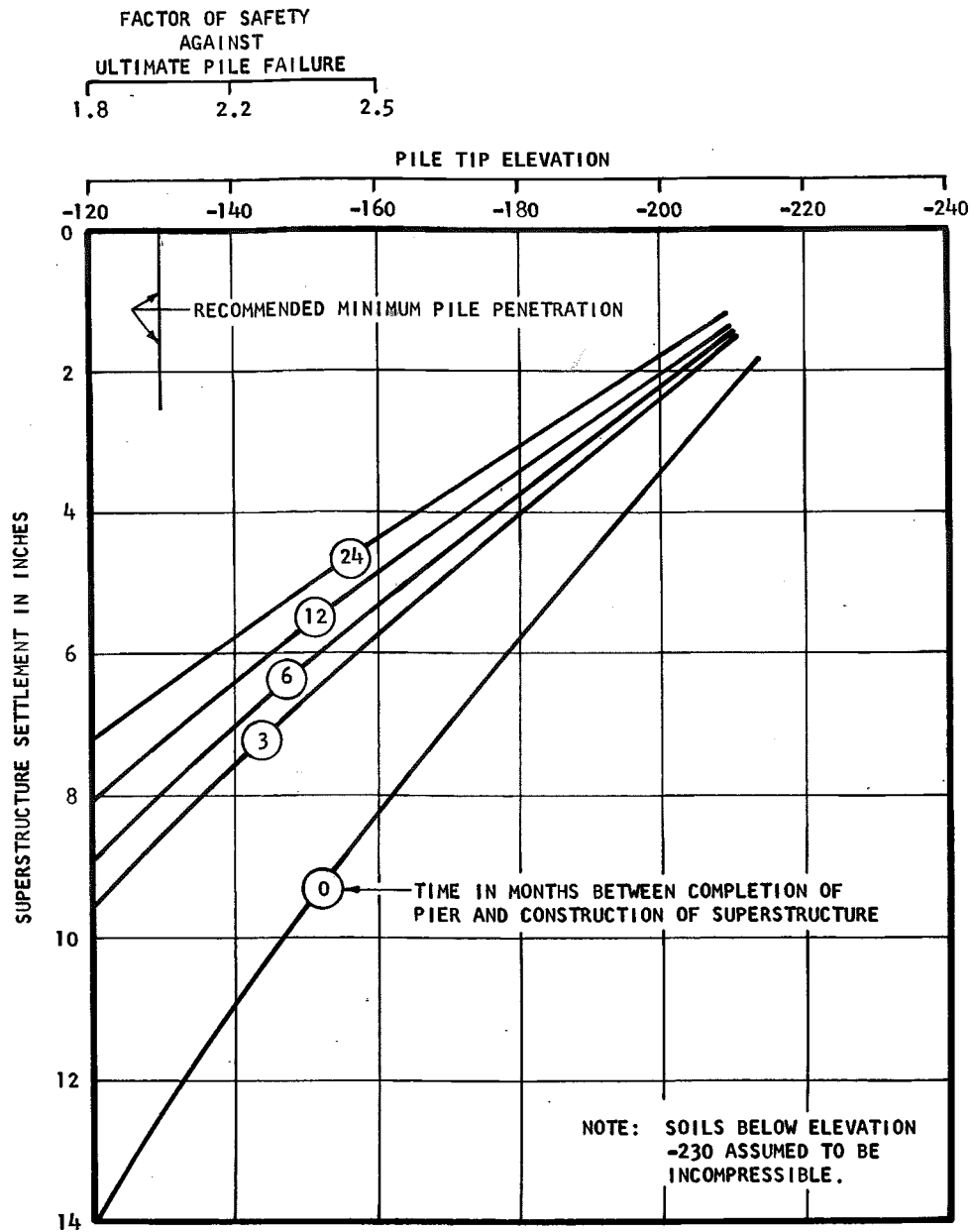
## SUMMARY OF FIELD AND LABORATORY OBSERVATIONS

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PLATE A-2

BY \_\_\_\_\_ DATE \_\_\_\_\_  
 BY \_\_\_\_\_ DATE \_\_\_\_\_  
 PLATE \_\_\_\_\_ OF \_\_\_\_\_

OREGON STATE HIGHWAY DEPT.  
 BY FBD DATE 1-22-62  
 CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_



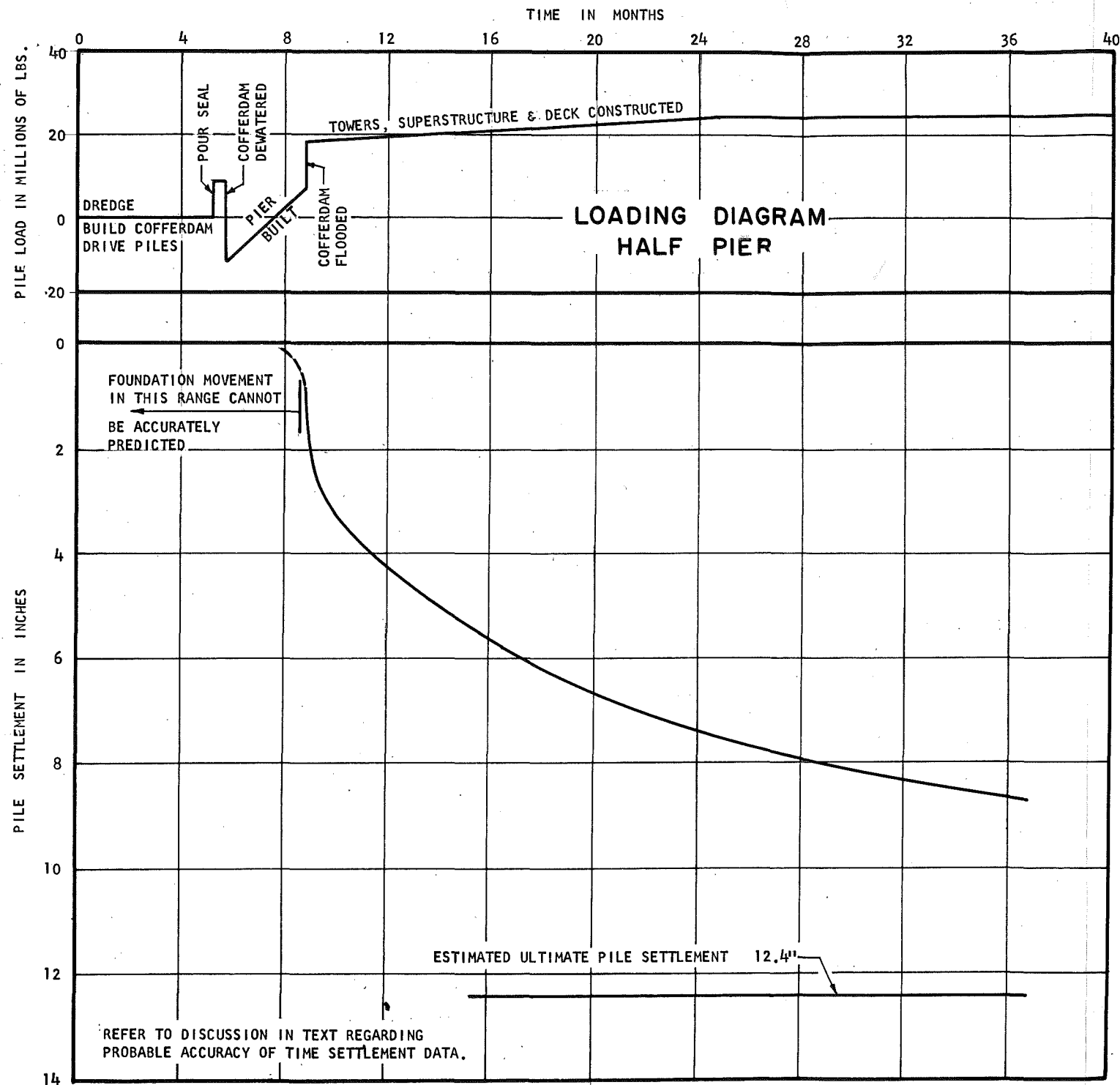
# ESTIMATED SUPERSTRUCTURE SETTLEMENT

DISPLACEMENT PILE FOUNDATION

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NOTE: TIME SCALE FOR THE LOADING DIAGRAM IS APPROXIMATE.  
THE TIME INTERVALS MAY VARY FROM THOSE SHOWN.



## TIME - SETTLEMENT STUDIES

WOOD PILE FOUNDATION  
PILE TIPS AT ELEVATION -130

**DAMES & MOORE**  
APPLIED EARTH SCIENCES

REVISIONS  
BY DATE  
BY DATE  
PLATE OF

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